Dearborn Environmental Consulting Group
Dearborn Chemical Company Limited
Subsidiary of W.R. Grace & Co.
3451 Erindale Station Rd.
P.O. Box 3060, Station A
Mississauga, Ontario, Canada L5A 3T5
Telephone: (416) 279-2222
Telex: 06-961145
Fax: (416) 279-0020

UTILIZATION POTENTIAL OF CFBC RESIDUE IN CONSTRUCTION APPLICATIONS

FINAL REPORT

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TABLE OF CONTENTS

		Page
LIST OF FIGURES		ii
LIST OF TABLES		iii
ACKNOWLEDGEMENTS		v
1.0	INTRODUCTION	1
1.1	Objectives	2
1.2	Report Organization	3
2.0	CONVENTIONAL AND AFBC CONCRETES: MATERIALS AND PERFORMANCE CHARACTERISTICS	4
2.1	Background	4
2.2	Fundamental Aspects of PC-Concrete	4
2.2.1	Concrete Making Materials	5
2.3	Summary	22
3.0	MATERIALS CHARACTERIZATION	23
4.0	MIX DESIGN/CONCRETE PERFORMANCE	31
4.1	Dalhousie PFA Reactivity	31
4.2	Lime Availability	33
4.3	Aggregate Requirements	36
4.4	Performance Characteristics	39
4.5	Summary	42
5.0	CONCLUSIONS	45
6.0	LITERATURE CITED	50

LIST OF FIGURES

Figure		Page
1	THE RELATION BETWEEN STRENGTH AND WATER/CEMENT RATIO OF CONCRETE	12
2	RELATIVE RATES OF STRENGTH INCREASE OF PLAIN CONCRETE AND FLY ASH CONCRETE PROPORTIONED BY SIMPLE REPLACEMENT OF CEMENT	12
3	PARTICLE SIZE DISTRIBUTION OF SPENT BED MATERIAL USED IN THE KECL STUDY	17
4	PARTICLE SIZE DISTRIBUTION OF PULVERIZED FUEL ASHES USED IN THE KECL STUDY	19
5	COMPRESSIVE STRENGTH OF CONTROL AND AFBC CONCRETE MIXES AS A FUNCTION OF CURING TIME	21
6	PARTICLE SIZE DISTRIBUTION OF CHATHAM CFBC ASH AND PULVERIZED FUEL ASHES USED IN STUDY	29
7	EFFECT OF AVAILABLE LIME CONTENT ON UNCONFINED COMPRESSIVE STRENGTH OF CHATHAM CFBC FLY ASH/DALHOUSIE PFA MORTAR MIXES	37
8	COMPARISON OF SETTING TIMES OF CONVENTIONAL PORTLAND AND AFBC NO-CEMENT CONCRETES	43

LIST OF TABLES

Table		Page
1	MATERIALS USED IN PC- AND AFBC-CONCRETES	4
2	CANADIAN PORTLAND CEMENT TYPES	5
3	GRADING LIMITS FOR FINE AGGREGATE	8
4	GRADING REQUIREMENTS FOR COARSE AGGREGATES	9
5	CONCRETE MIX PROPORTIONS	10
6	PHYSICAL PROPERTIES OF TEST AGGREGATE	14
7	CHEMICAL CHARACTERISTICS OF THE TVA AFBC SPENT BED RESIDUE	16
8	CHEMICAL AND PHYSICAL CHARACTERISTICS OF PULVERIZED COAL ASHES SUPPLIED BY TVA FOR THE KECL STUDY	18
9	COMPARISON OF CHATHAM CFBC AND TVA 20-MW AFBC ASH COMPOSITIONS	24
10	HYDRATION TESTS ON CHATHAM CFBC ASH	25
11	CHEMICAL AND PHYSICAL CHARACTERISTICS OF PULVERIZED COAL ASHES USED IN CHATHAM CFBC AND KECL AFBC ASH UTILIZATION STUDIES	27
12	SUMMARY OF POZZOLAN CHARACTERIZATION OF KINGSTON AND DALHOUSIE PFA	28
13	COMPARISON OF PARTICLE SIZE DISTRIBUTION OF COARSE AGGREGATES USED IN CONCRETE MIXES	30
14	COMPARISON OF PERFORMANCE OF DALHOUSIE AND KINGSTON PFA USED IN CHATHAM AFBC MORTAR MIXES	32
15	EFFECTS OF LIME ADDITION ON UNCONFINED COMPRESSIVE STRENGTHS OF CHATHAM CFBC ASH BASED CONCRETE MIXES	34
16	EFFECT OF LIME ADDITION ON COMPRESSIVE STRENGTH OF CHATHAM CFBC FLY ASH MORTAR MIXES UTILIZING DALHOUSIE PFA	35
17	SIZE DISTRIBUTION AND AGGREGATE REPLACEMENT POTENTIAL OF CHATHAM CFBC ASHES	38
18	EFFECT OF COARSE AGGREGATE CONTENT ON WATER/EFFECTIVE CEMENT RATIO AND WORKABILITY	40

19	PERFORMANCE CHARACTERISTICS OF PRELIMINARY CHATHAM CFBC CONCRETES	41
20	PROPORTION OF MATERIALS USED TO PREPARE NO-CEMENT CONCRETES	46
21	TYPICAL COSTS FOR CONCRETE AND CONCRETE-MAKING MATERIALS IN ATLANTIC CANADA (1988)	46

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1.0 INTRODUCTION

Atmospheric fluidized bed combustion (AFBC) technology is rapidly gaining acceptance for the combustion of high sulphur content solids fuels. In the combustion process, coarsely crushed fuel, together with limestone, is suspended in the combustion chamber by the injection of fluidizing air from below the bed. During combustion, the limestone is calcined to CaO, which combines with the SO_2 released during combustion and oxygen to form calcium sulphate. The control of sulphur emissions is accomplished within the combustion chamber, which eliminates the need for add-on flue gas scrubber technology.

Circulating fluidized bed combustion (CFBC) is a recent development which is particularly suited to large scale applications in Eastern Canada since it is capable of combusting high sulphur coals while still achieving strict air emission requirements. The process is currently being demonstrated at the Energy, Mines and Resources Canada funded Chatham, New Brunswick project. One limitation of the sulphur capture process appears to be the large quantities of solid wastes that are produced. Thus, the most serious environmental concerns associated with this technology are likely to be the handling, management and disposal of the resulting solid waste.

The physical, chemical, geotechnical and handling properties of CFBC wastes from a laboratory scale combustor were recently characterized by Dearborn as part of a Canadian Electrical Association study (1). The combustion residues were found to be composed mainly of anhydrous calcium sulphate, calcium oxide, calcined shales and clays from the ash-forming minerals in coal, trace elements and calcium sulphide. In general, the wastes were largely composed of particles in the silt size ranges and had a very narrow size distribution. As a result, special efforts would be needed to reduce the impacts of the disposal of these wastes in the environment. The management of residues from this advanced coal combustion process will be an area of concern and interest for proponents of the technology. Before CFBC technology can achieve widespread commercial acceptance, cost effective ash disposal practices must be developed.

Alternately, the barrier of residue handling and disposal could be overcome by reducing the amount of residue to be disposed of by developing ways to utilize the waste rather than merely disposing of it. The implementation of such a program would reduce the volume of wastes to be handled and disposed of thereby reducing operating costs, and in some cases, may represent a source of revenue. A residue utilization program could favourably effect the economics of CFBC projects such as at Chatham, New Brunswick.

Unfortunately, the uses of residues from circulating fluidized bed combustion have not yet been widely investigated. However, based on previous characterization studies, the most promising applications for high-calcium AFBC wastes are considered to be:

- in low-strength concrete;
- in soil cementing and stabilization;
- as an aggregate/filler in asphaltic concrete;
- as a source of lime for waste stabilization, land reclamation and waste neutralization;
- as a source of agricultural lime and nutrient values.

In an eastern Canadian context, the utilization of CFBC residues in construction applications seems most promising and thus, it is the subject of this investigation.

1.1 Objectives

The overall objective of the study was to investigate the potential of utilizing circulating fluidized bed combustion residues in construction applications. More detailed objectives include:

o review relevant materials and performance characteristics of conventional and AFBC concretes;

o characterize CFBC residues in terms of pozzolanic properties;

- investigate mixing properties of the CFBC residue;
- determine optimum CFBC residue/PFA mixtures;
- prepare a detailed technical report describing the results of the study including recommendations for further investigation.

1.2 Report Organization

This report is composed of three sections. The first consists of a review of relevant literature on the theory and chemistry of cementitous materials. The second section of the report presents the methods used and results of the characterization of the Chatham CFBC residue. This section also describes the overall flow of the materials testing program. The final section of the report describes the test program and results from the concrete mix design investigations as well as conclusions arising from this study.

2.0 CONVENTIONAL AND AFBC CONCRETES: MATERIALS AND PERFORMANCE CHARACTERISTICS

2.1 Background

Concretes are composite materials that can be made from a wide variety of materials. Their characteristic property is that they are composed of hard, strong particles of aggregate (sand, stone or metal) bound together by a cement that can be mixed with the aggregate in a semi-fluid state and that will then harden into a stable solid. Cements of this kind can be thermo-setting plastics, asphalt, sulphur, or most frequently one of a number of hydraulic inorganic materials such as Portland cement.

The vast majority of concretes used in normal construction practice, are made from Portland cement, stone and sand. They are so common that Portland cement concrete (PC-concrete) is normally referred to as only "concrete".

The materials made from AFBC residues by Rose et al (2 - 5) at the Kentucky Energy Cabinet (termed AFBC-concrete in this report) are distinguished from PC-concretes by the use of AFBC residue both to replace the fine aggregate, and to act, in conjunction with fly ash, as the cementing binder. The compositional differences between AFBC- and PC-concretes are summarized in Table 1.

2.2 Fundamental Aspects of PC-Concrete

To establish a basis for comparison, the relevant aspects of the preparation, nature and properties of PC-concrete must be considered. It is not possible in this report to provide a full description of PC-concrete or other related materials. For such information reference can be made to standard texts (6,7). Certain features of concretes must be understood if a critique of AFBC-concrete is to be made. These are discussed in the following paragraphs.

TABLE 1 MATERIALS USED IN PC- AND AFBC-CONCRETES

Material used in PC-Concrete	Corresponding Material in AFBC-Concrete
Portland Cement	A mixture of fly ash and AFBC spent-bed residue.
Stone	Stone
Sand	Coarse particle component of AFBC spent-bed residue.
Water to hydrate Portland cement and make the mix workable.	Water: (1) to pre-hydrate the AFBC spent-bed residue. (2) to support pozzolanic reactions and make the mix workable.

2.2.1 Concrete Making Materials

Portland cements. Portland cements are composed of a complex mixture of calcium silicates, aluminates and ferrites in the form of a finely ground clinker, mixed with a small quantity of calcium sulphate (usually gypsum). Portland cement is made by burning a mixture of limestone, clay or shale, mill-scale and sand in a rotary kiln at 1500 to 1600°C. The resulting clinker is allowed to cool rapidly and is then finely ground with an addition of approximately 5% of gypsum. Depending upon its application and chemical composition, Portland Cement is sold in Canada and the USA under one of five types, the nomenclatures of which are given in Table 2. Portland Cement in Canada is governed by CSA Standard CAN3-A5-M83 of the Canadian Standards Association.

TABLE 2 CANADIAN PORTLAND CEMENT TYPES

Name	Type
Normal Portland Cement	10
Moderate Portland Cement*	20
High Early-Strength Portland Cement	30
Low Heat of Hydration Portland Cement	40
Sulphate Resistant Portland Cement	50

*Moderate with respect to heat of hydration or sulphate resistance.

When Portland cement is mixed with water to form a paste, a sequence of reactions commences that ultimately results in its hardening into a strong solid mass. When paste and sand are mixed in the un-hardened state and allowed to set, the combination is termed a mortar. When stone is included, the composite material formed is a concrete. Unlike a number of other inorganic cements, for example calcium sulphate plaster (stucco), hardened Portland cement is stable to continued exposure to water, and is thus termed an hydraulic cement. In all but the most arid climates, hydraulic cement is required for all exterior exposure.

The reactions responsible for hardening in Portland cement are complex, the following being the most important:

$$Ca_3Al_2O_6 + 6H_2O \longrightarrow Ca_3Al_2(OH)_{12}$$
$$Ca_2SiO_4 + xH_2O \longrightarrow Ca_2SiO_4 \cdot xH_2O$$
$$Ca_3SiO_5 + xH_2O \longrightarrow Ca_2SiO_4 \cdot (x-1)H_2O + Ca(OH)_2$$

The calcium silicate hydrate, $Ca_2SiO_4 \cdot xH_2O$ is termed CSH for convenience and is the principal component responsible for the cementing action of Portland cement. In most Portland cement, tricalcium silicate (Ca_3SiO_5) is the main component and hence the production of $Ca(OH)_2$ during hydration is a common and important feature of the process. Calcium sulphate in Portland cement is not used as a cementing component. It's function is to regulate the rate of early setting and prevent the hydration reactions from occurring too rapidly to permit placement of the concrete mass. In this role and during the hardening process, calcium sulphate reacts with aluminate ions and calcium hydroxide to form sulpho-aluminates. These are important contributors to early strength and to the expansion of cement paste during hydration.

Pozzolans. A CSH binder can be formed directly from sources of reactive silica by reaction with calcium hydroxide and water. Such reactions are termed pozzolanic, and are usually a feature of the use in concrete of pozzolans such as volcanic ash, thermally treated clays or shales, or fly ash. Of these, fly ash is the most important in modern concrete practice.

In a mixture of pozzolan, calcium hydroxide and water, strength develops slowly and usually to a lesser degree than is the case with Portland cement concrete. However, pozzolanic concretes ultimately develop great strength and durability. Some of the oldest standing structures, built by ancient Greek and Roman civilizations, were made using lime and volcanic ash as cements.

The pozzolanic reactions, especially when fly ash is the source of silica are exceptionally complex and poorly understood. For the purposes of comparison with Portland cement the following reaction is sufficient to illustrate the principal features of pozzolanic activity:

$$xCa(OH)_2 + ySiO_2 + zH_2O \longrightarrow CSH$$

The important aspect of this reaction is that calcium hydroxide is consumed by the pozzolan to produce CSH. This is distinct from the hydration of tricalcium silicate in Portland cement, in which $Ca(OH_2)$ is formed. Because of these factors, pozzolans can be used in conjunction with Portland cement where the cement produces lime and the pozzolan consumes it. In both cases CSH is formed. In general, fly ash is used with Portland cement in construction concrete, and with lime in soil-cementing compositions. The use of fly ash with cement has been extensively studied and detailed discussions of the use of fly ash in concrete can be found in a number of recently published monographs (8,9). Pozzolans for use in concrete are subject to CSA specification CAN/CSA A23.5-M86.

Aggregates. Much of the volume of normal construction concrete is provided by aggregate. Stone and sand are used, and may come from many sources, requiring only that they meet specifications for strength, freedom from contamination by harmful materials, and durability. In most urban areas of Canada, established concrete aggregates have been quarried for many years and provide reliable sources of construction materials. Depending upon the type of concrete to be produced, the maximum size of the coarse aggregate will normally be specified

with a standard gradation of sizes grading down to fine sized particles. The principal of grading aggregates is based upon the packing of spheres of different sizes which allows the small spheres to locate themselves in the voids between closely packed larger spheres. The purpose is to fill as much space with inexpensive aggregate as possible, while leaving the more costly cement paste to coat all aggregate particles and fill the spaces between sand grains. Aggregate grading is also important to the workability of fresh concrete. A mix with too little fines will be hard to place without enclosing air voids into the mass. A mix in which the gradation is dominated by fine particles will be sticky to handle, segregate and shrink during curing. Grading limits for concrete aggregates are shown in Tables 3 and 4. As with other concrete-making materials, aggregates are regulated in Canada by CSA specification CAN3-A23.1-M77.

TABLE 3 GRADING LIMITS FOR FINE AGGREGATE

Sieve Size	Total Passing Sieve % by Mass
10 mm	100
5 mm	95 - 100
2.5 mm	80 - 100
1.25 mm	50 - 90
630 m	25 - 65
315 m	10 - 35
160 m	2 - 10

TABLE 4 GRADING REQUIREMENTS FOR COARSE AGGREGATES

	Nominal Size of Aggregate mm	Total Passing Each Sieve*, Per Cent by Mass										
		112 mm	80 mm	56 mm	40 mm	28 mm	20 mm	14 mm	10 mm	5 mm	2.5 mm	1.25 mm
Group I	40—5	—	—	100	95—100	—	35—70	—	10—30	0—5	—	—
	28—5	—	—	—	100	95—100	—	30—65	—	0—10	0—5	—
	20—5	—	—	—	—	100	90—100	—	25—60	0—10	0—5	—
	14—5	—	—	—	—	—	100	90—100	45—75	0—15	0—5	—
	10—2.5	—	—	—	—	—	—	100	85—100	10—30	0—10	0—5
Group II	80—40	100	90—100	25—60	0—15	—	0—5	—	—	—	—	—
	56—28	—	100	90—100	30—65	0—15	—	—	0—5	—	—	—
	40—20	—	—	100	90—100	25—60	0—15	—	0—5	—	—	—
	28—14	—	—	—	100	90—100	30—65	0—15	—	0—5	—	—
	20—10	—	—	—	—	100	85—100	—	0—20	0—5	—	—
	14—10	—	—	—	—	—	100	85—100	0—45	0—10	—	—
	10—5	—	—	—	—	—	—	100	85—100	0—20	0—5	—

*Sieves shall meet the requirements for woven wire cloth testing sieves given in CGSB Standard 8-GP-2M, Sieves, Testing, Woven Wire, Metric.

Notes:

(1) Group I comprises combined aggregate gradings most commonly used in concrete production. Group II provides for special requirements i.e. gap grading, pumping, etc.

(2) It is not recommended that the 40—5 mm and 28—5 mm gradings in Group I be placed in stockpile or batched to the mixer in one size, owing to the segregation of the aggregate that may occur.

(3) If one of the nominal sizes of aggregate shown in Group I does not by itself meet the special requirements of the work under consideration, an acceptable overall grading may be obtained by batching with one or more of the standard sizes shown in Group II.

Admixtures. A number of proprietary admixtures are used in small quantities to alter the properties of fresh concrete. The most important are, set-control agents, such as retarders or accelerators. These allow the concrete to remain workable for preselected periods of time to permit its transportation or placement under particular construction conditions. Water reducing agents and superplasticizers are used to give fluidity to high strength concretes of low water content or to make normal concrete flow into regions of congested reinforcing steel. Air-entraining agents are used to introduce controlled quantities of air bubbles into a mix as part of durability control.

Reinforcement. Most structural concrete incorporates an array of steel rods that provide reinforcement. Concrete is a brittle material with little tensile strength. The use of reinforcement allows it to be used in conditions where tensile forces must be resisted or where ductility is required under stress. Many of the durability problems associated with concrete involve in one way or another corrosion of reinforcing steel. Any new material for use in concrete must be carefully considered in regard to the potential to induce corrosion before it is used in a reinforced concrete structure.

Concrete. Concrete is made from the above materials by mixing them, usually at a mixing plant in proportions that are selected largely on the basis of an accumulated experience. The proportions are determined by two technical criteria:

- the required minimum workability or flow of the unhardened mix for placement purposes;
- the desired level of strength required for the structure in question.

The process of meeting these criteria at a minimum cost is termed mix design or mix proportioning. Mix proportions are shown in Table 5 for two PC-concrete compositions each with a design-strength of approximately 35 MPa (5000 psi) at 28 days. Mix 1 is a typical PC-concrete while Mix 2 is an equivalent fly ash concrete.

TABLE 5 CONCRETE MIX PROPORTIONS (% by mass)

Component	PC-Concrete	Fly Ash Concrete
Water	6.5	5.5
Cement	13.0	8.9
Fly ash	-	4.4
Sand	34.6	34.6
Stone	45.9	45.9

As noted above, to design a concrete mix requires attention to both workability and strength development. Because concrete is poured from the mixer and worked into its final site (either within forms that are removed after hardening, or as a horizontal slab) it must be fluid enough that it can be formed and vibrated into place. It is unfortunate that for most ordinary concrete, the quantity of water that is required to hydrate the cement (about 23% by mass of Portland cement), is too little to give workable fluidity to the mix. Some fluidity can be obtained by reducing the stone content and increasing the fine sand and cement, however this is at the cost of other desired features. Thus, unless special admixtures are used, the concrete must contain an 'excess' of water. After hydration, this water remains leaving voids in the hardened concrete.

Like other brittle materials, much of the strength of concrete is determined by the density of its microstructure. Voids are sources of weakness and reduce the average strength of concrete. Experience has shown that the strength of hardened concrete, usually specified as that attained after 28 days of curing, can be related to the ratio of the mass of water to the mass of cement used in the fresh mix (w/c). Figure 1 shows the relationship between w/c and strength in the form developed by Abrams. When a pozzolan is used, similar relationships apply in which a contribution by the pozzolan to strength development accommodated (8).

In practice, workability is determined at the time of mixing by measuring the slump of fresh concrete in accordance with specifications. Slump is a measure of the consistency or fluidity of concrete where low slumps correspond to a stiff consistency. The test provides a measurement of the deformation of an unsupported cone of wet concrete due to forces of gravity after a specified time period. If the quantity of cement and the w/c ratio have been correctly selected, concrete will be easily placed, and unless it is subjected to unusual conditions during curing such as freezing, it will normally cure to a reliable strength. The rate of curing will vary with the materials used, and, different sources of cement will differ somewhat in curing rate. In most concrete practice, strength at 28 days is the criterion of acceptance. In

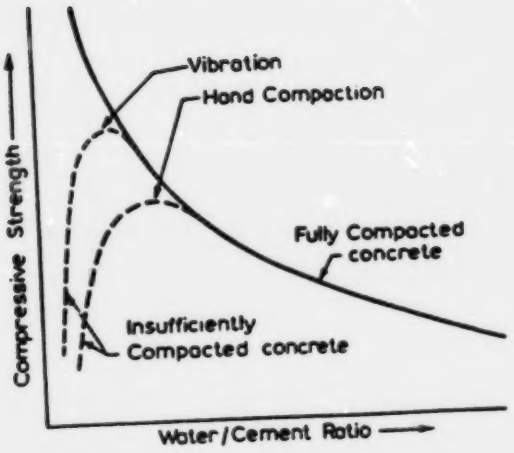


FIGURE 1 THE RELATION BETWEEN STRENGTH AND WATER/CEMENT RATIO OF CONCRETE

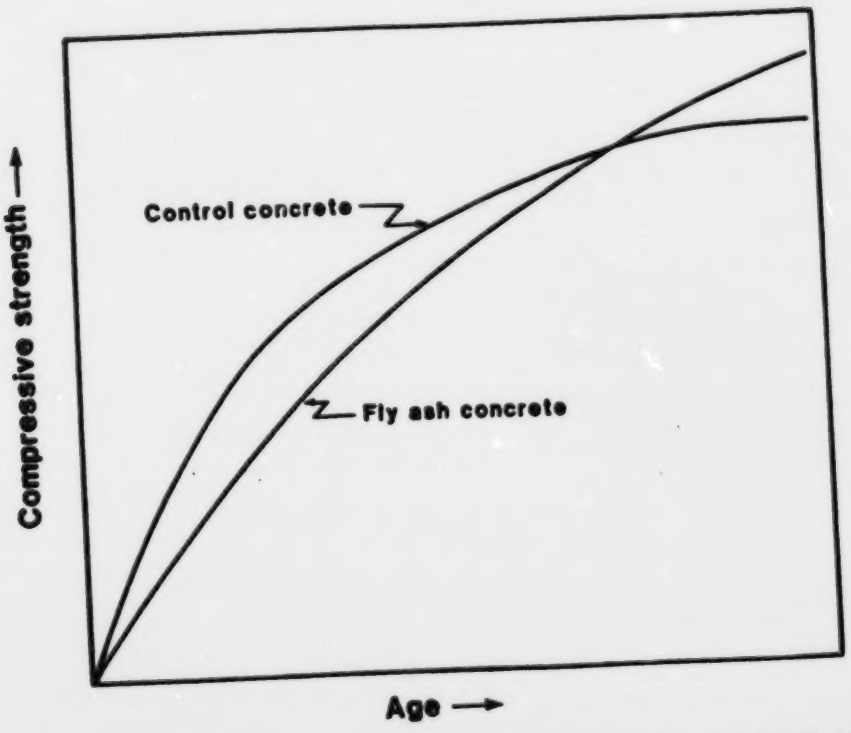


FIGURE 2 RELATIVE RATES OF STRENGTH INCREASE OF PLAIN CONCRETE AND FLY ASH CONCRETE PROPORTIONED BY SIMPLE REPLACEMENT OF CEMENT

special circumstances, in order to meet construction demands, strength at earlier ages may be required. Figure 2 shows the normal form of strength development with curing time relationship in PC-concrete and in fly ash concrete where fly ash has been used to replace an equal volume of cement.

Much of the confidence placed in the performance of concrete is a consequence of the extent to which quality controls are placed on the materials used, either as a result of specifications and standards or by the manufacturer of cement. Consequently there is great resistance in the construction industry to the introduction of radically new materials unless their performance has been extensively proven and shown to be reliable and predictable. It should be noted, in regard to practical construction, that the cost of concrete is small relative to the labour costs of its placement. Thus, although savings in materials costs are desirable, they will rarely be accepted by the industry if there is more labour required in the production process, or if there is a risk that failure to perform may require removal and replacement of concrete in a construction project. For these reasons the industry is normally slow to accept new materials. However, because of the major technical improvements in concrete over the past ten years, many new approaches to concrete construction and concrete materials have been accepted.

AFBC-concretes. As is apparent from Table 1, the AFBC-concretes developed at the Kentucky Energy Cabinet, are made from markedly different starting materials than are PC-concrete. The most important differences are that:

- they contain no Portland cement; the binder is totally a pozzolanic cement made from fly ash and lime (from the AFBC residue);
- they contain no fine aggregate (sand); this is provided by the unreacted part of the AFBC residue.

They are thus unique construction materials.

The process of making AFBC-concretes also differs from that used for PC-

concretes in that the mixing must be conducted in two stages:

- a prehydration stage;
- a concrete mixing stage.

The principal details of the work conducted by KEC were reported by Rose et al in September, 1986 (2), a summary of this work was subsequently presented by Jones, Bland, Rose and Jarrett (5). The main points from this work are discussed in this report, however, for details the reader is referred to the above noted reports and others in the literature (2 - 5).

Materials. The concrete mix proportions used for the control comparison mix are given in Table 1. The study by Rose et al (2) considered one source (but several samples) of AFBC spent-bed residue supplied by TVA from the Shawnee 20 MW AFBC pilot plant, Paducah, Kentucky. Characteristics of these samples are shown in Table 7, with the corresponding particle size distribution shown in Figure 3. Two pulverized coal fly ashes also from TVA sources (Shawnee, and Kingston plants) were used. Details of relevant properties are given in Table 8, and particle size characteristics are shown in Figure 4. A coarse limestone aggregate was used with the physical properties shown below (Table 6) and as noted above no sand was used in the AFBC-concretes.

TABLE 6 PHYSICAL PROPERTIES OF TEST AGGREGATE

SSD Specific Gravity*	2.70
Absorption (%)	1.28
Los Angeles Abrasion Loss	20%
Grain Size (% Passing)	
1"	100
0.75"	84.9
0.5"	62.6
#4 sieve	26.6

*SSD = Saturated Surface Dry

Prehydration of AFBC spent-bed residue. It is well established (10) that

AFBC spent-bed residues comprise particles that are not homogenous. They consist of a core of CaO surrounded by the products of reaction between CaO and the sulphur oxides from combustion usually in the form of anhydrous $CaSO_4$. Both of these are reactive with water to form $Ca(OH)_2$ and $CaSO_4 \cdot 2H_2O$, respectively, by exothermic reactions that involve molar volume increases. These are summarized below:

$$CaO + H_2O \longrightarrow Ca(OH)_2 \qquad [1]$$

$$CaSO_4\ 0\ 5\ H_2O + 1.5\ H_2O \longrightarrow CaSO_4 \cdot 2H_2O \qquad [2]$$

Reaction [1] will result in the release of 15.6 $kcal.mole^{-1}$ and a molar volume expansion of 97%. Similarly the complete hydration of calcium sulphate, reaction [2] will release 4.22 $kcal.mole^{-1}$ and a molar volume expansion of 37%. In a mixture, such as AFBC residue, the heat released will be dominated by the formation of calcium hydroxide from CaO.

The volume increases upon hydration, unless completed in the very early stages of setting of a concrete, will cause fracturing and reduced strength in the finished materials (2,3,11). From the practical perspective of using AFBC wastes as construction materials, it has been found necessary to permit these reactions to occur as a preliminary step before their inclusion in a concrete or soil cementing mixture.

The prehydration process was examined at KECL in an optimization study, the specific details of which are given by Rose et al (2). The final optimal mix was found to require the addition of 10% of water. This reduced the subsequent temperature rise during concrete mixing to less than two degrees above that reached by the control mix (approximately 22°C), and at the same time achieved maximal compressive strength development in the final concrete.

The optimum conditions for prehydration should be considered as potentially source- and sample-specific. The use of 10% water is less than the stoichiometric requirement for hydration of the bed-residue used by KECL. Also, the optimal quantity will depend on the relative proportions of calcium oxide and sulphate in the residue and probably on particle size. It is necessary to

TABLE 7 CHEMICAL CHARACTERISTICS OF THE TVA AFBC SPENT BED RESIDUE.

Parameter	Spent Bed	
	Sept. 1986 Study	Sept. 1988 Study
CHEMICAL COMPOSITION (wt%)		
Moisture	0.01	0.17
Ash	97.59	95.70
Carbon (Total)	0.44	0.31
Carbon (Mineral)	0.30	0.16
Sulfur	14.45	12.80
MINERAL ANALYSIS (wt%)		
SiO_2	4.25	4.54
TiO_2	0.11	0.07
Al_2O_3	1.49	1.36
Fe_2O_3	2.68	1.80
CaO	53.85	55.82
MgO	1.24	2.08
K_2O	0.18	0.20
Na_2O	0.07	0.28
P_2O_5	0.09	0.05
SO_3	36.13	32.00
FORMS OF CALCIUM (wt%)		
CaO	26.17	32.18
$CaSO_4$	61.41	54.40
$CaCO_3$	2.56	1.33
SPECIFIC GRAVITY	2.67	NA
ABSORPTION (%)	3.23	NA

NA = Not Available

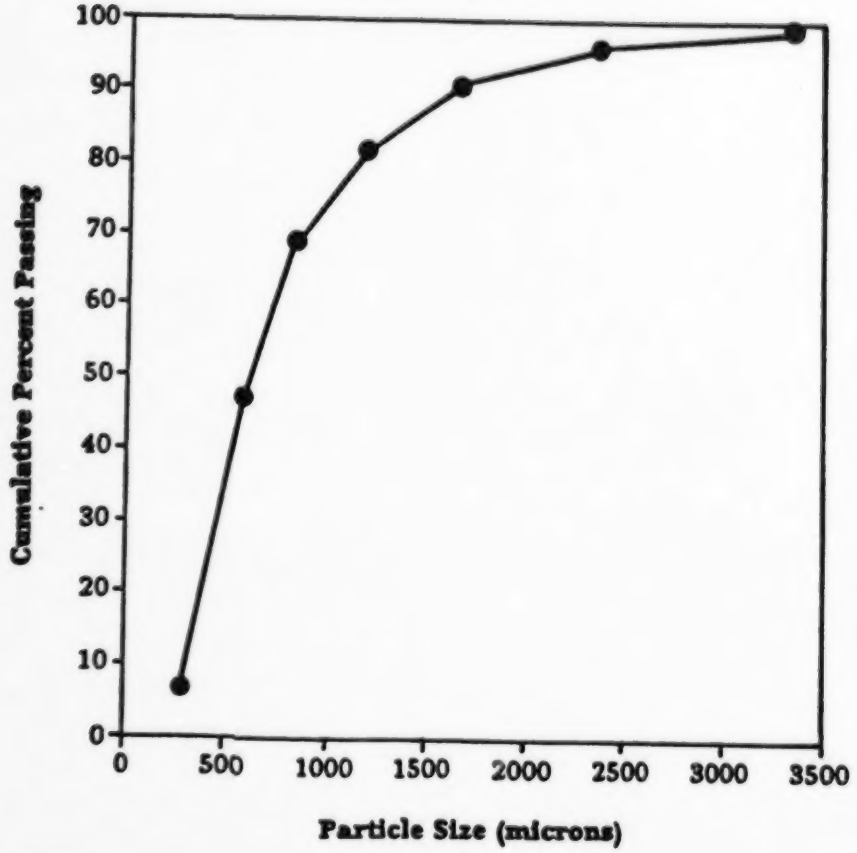


FIGURE 3 PARTICLE SIZE DISTRIBUTION OF SPENT BED MATERIAL USED IN THE KECL STUDY

TABLE 8 CHEMICAL AND PHYSICAL CHARACTERISTICS OF PULVERIZED COAL ASHES SUPPLIED BY TVA FOR THE KECL STUDY

	Shawnee	Kingston
Moisture (wt %)	0.35	0.38
Ash (wt %)	93.10	96.33
$Carbon_{total}$ (wt %)	5.89	2.30
Sulfur (wt %)	0.38	0.28
Loss on Ignition (wt %)	6.77	3.42
Specific Gravity (wt %)	2.05	NA
Ash Analysis (wt %)*		
SiO_2	54.51	51.20
TiO_2	1.75	1.40
Al_2O_3	32.19	30.58
Fe_2O_3	5.05	8.49
CaO	2.22	2.10
MgO	1.16	1.52
K_2O	2.19	3.13
Na_2O	0.51	0.32
P_2O_5	0.23	0.59
SO_3	0.19	0.18
	100.00	99.61

* 750°C Ash
NA = Not analyzed.

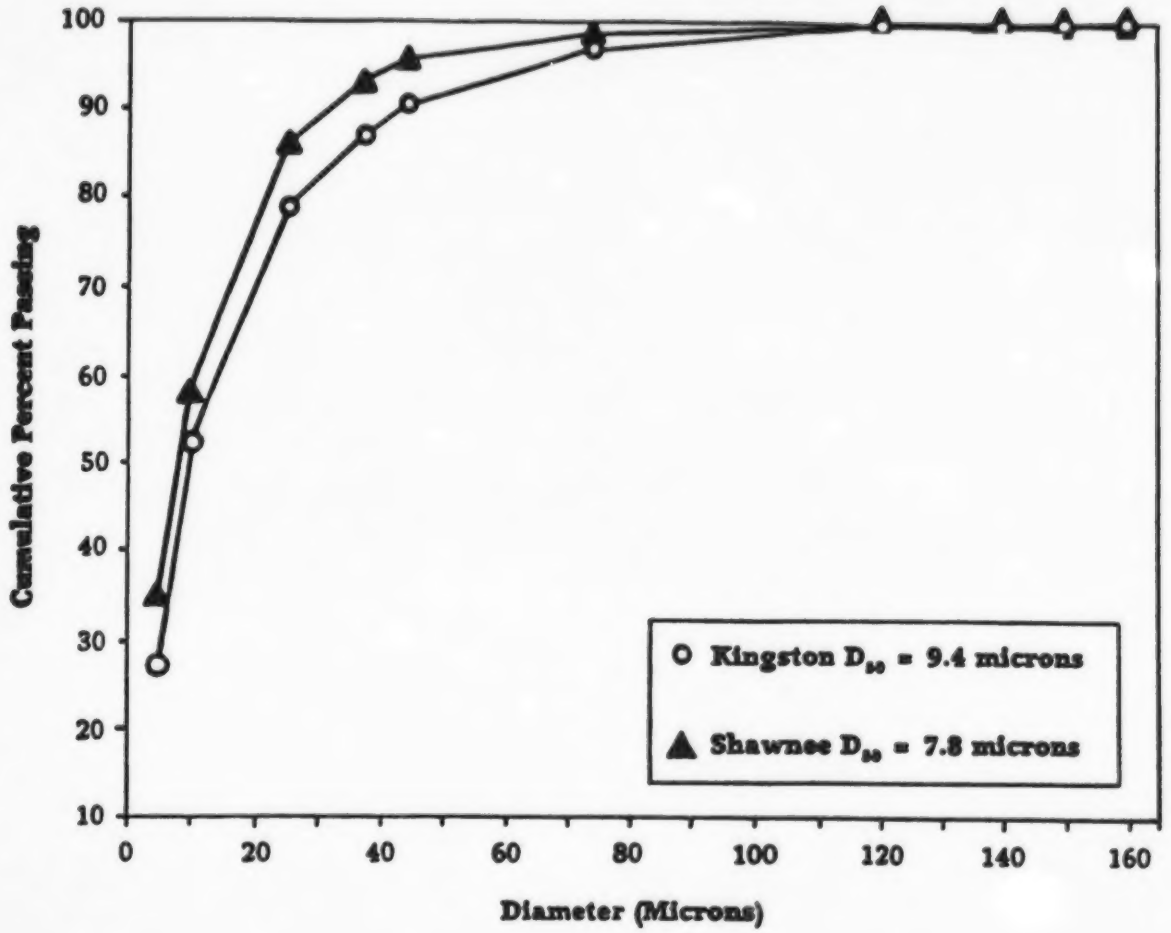


FIGURE 4 PARTICLE SIZE DISTRIBUTION OF PULVERIZED FUEL ASHES USED IN THE KECL STUDY

establish the quantity of water required for prehydration of each source of AFBC residue and this quantity may vary with changes in the combustion conditions. This may make the use of AFBC residue difficult. The establishment of suitable conditions and quality control methods for AFBC-concretes will require investigation of the nature and sample dependence of the prehydration step at an early stage prior to commercialization.

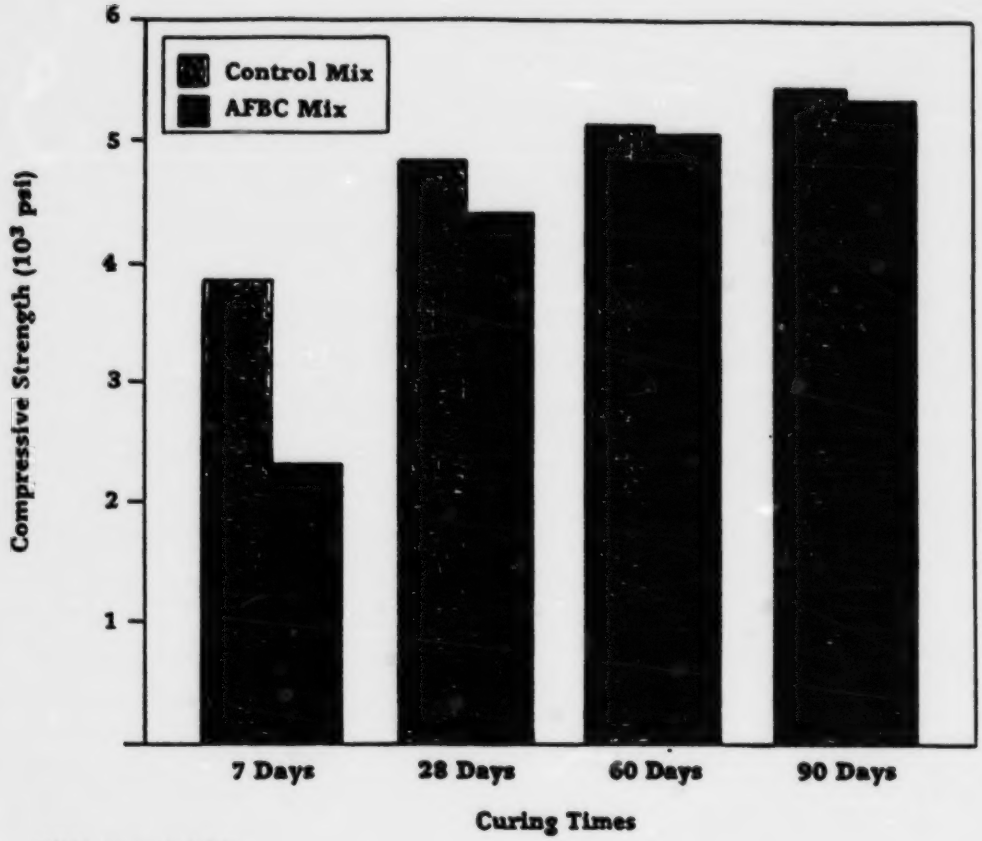
A similar optimization investigation was conducted at KECL to establish the proportions to be used for the AFBC-concrete (2). A ratio of 1 part fly ash, 2.1 parts spent bed ash and 3.4 parts coarse limestone was established to produce a workable concrete at 4 to 5 inches of slump, giving the maximum compressive strength. The performance characteristics of this concrete were examined and reported, in comparison to a conventional PC-concrete (Table 1).

Compressive strength development of the resulting concretes, over a 90 day curing period, is shown in Figure 5. It is clear that the rate of strength development of the AFBC-concrete was slower than for the control concrete, although close to 90% of control strength was reached by 28 days and nearly equal strength was reached by 60 days.

The modulus of elasticity of the AFBC-concrete was 19 GPa compared to 40 GPa for the control, indicating a less brittle and rigid concrete. This is probably a result of the low hardness of the unreacted AFBC residue in comparison with the sand used in the control concrete.

Setting times were slow in comparison with the control, 15.8 hours versus 4.8 hours, respectively, for initial-set, and 24.8 hours versus 7 hours for final-set, respectively. Increased setting times are common with PFA containing concretes. In addition, the high $CaSO_4$ content probably also retarded setting. These may have a serious effect on placement and form removal procedures for the AFBC-concretes. Jones et al (5) have suggested that chemical admixtures may be required to accelerate setting.

Other aspects of the behaviour of AFBC-concrete were noted as follows:



1000 psi = 6.9 MPa

FIGURE 5 COMPRESSIVE STRENGTH OF CONTROL AND AFBC CONCRETE MIXES AS A FUNCTION OF CURING TIME

- early shrinkage due to bleeding of the order of 0.41% occurred, followed by expansion to 0.33% by 90 days;

- permeability to water at early ages was much higher than the control concrete. The permeability reduced after curing and it appears that the values measured at early ages were adversely affected by the immature nature of the AFBC-concrete.

Slabs and blocks have been made from AFBC-concrete mixes and durability studies are in progress.

2.3 Summary

AFBC-concretes are interesting materials with a definite potential as a means to utilize AFBC-residues. However, much development work is required to establish them as construction materials. The following issues are must be examined:

- the influence of material variations on the properties of AFBC-concretes;

- the extent to which the properties are source specific;

- the nature of the hydration products in the cement paste fraction of AFBC-concrete;

- the durability of AFBC-concretes in an extended range of conditions.

Overall when compared with the many uses for AFBC residues that have been examined (11), the AFBC-concretes appear, at the present stage of their development to, offer a good potential for commercial application.

3.0 MATERIALS CHARACTERIZATION

The materials characterization included the chemical and physical characterization of the Chatham CFBC ash and the Dalhousie PFA. Hydration testing of the Chatham CFBC ash was also included. The chemical composition of the Chatham CFBC baghouse and bed material are shown in Table 9. Also included in Table 9 for comparison are the compositions of AFBC ashes from the TVA 20 MW AFBC pilot plant. As previously mentioned, the TVA ashes have been extensively studied by the KECL for production of concretes (2-5) and the Chatham CFBC ash appears to be similar to the baghouse and char ashes from the TVA 20 MW AFBC. In terms of the forms of calcium, the Chatham CFBC ash contains approximately 27% CaO compared with a range of 26-32% CaO in the spent bed ash from the TVA 20 MW AFBC pilot plant. It is the CaO which reacts with the PFA to form pozzolanic reaction products in the concrete. It is important to note that the Chatham ashes contain a lower amount of $CaSO_4$ than the TVA 20 MW AFBC spent bed and this may be a contributing factor to reducing the setting times associated with the AFBC concretes.

A series of hydration tests was conducted on the Chatham CFBC ash. The objective of the hydration was to convert the CaO to $Ca(OH)_2$. Previous results (2-5) have shown that hydration of the AFBC ash is critical in controlling temperature excursions and volume expansions during subsequent handling or use. The results from the hydration of the Chatham ash are shown in Table 10. Lime [$Ca(OH)_2$] concentrations were determined by thermogravimetric analysis (TGA). $CaSO_4 \cdot 2H_2O$ was not found during these analyses. Complete conversion of the CaO to $Ca(OH)_2$ was not achieved. The highest conversion achieved was 76%, corresponding to 24.96% $Ca(OH)_2$ in the hydrated solid. The use of hot water and pre-heated ash were effective in generating steam, but did not prove effective in increasing the rate of conversion. This is surprising, since it was thought that steam may help penetrate the $CaSO_4$ shell surrounding the CaO. Previous studies with TVA 20 MW spent bed have shown CaO conversions of about 95%. However, the particle size distribution of the TVA material was much larger, facilitating more cracks, allowing water (or steam) penetration. The small particle size distribution of the Chatham

TABLE 9 COMPARISON OF CHATHAM CFBC AND TVA 20 MW AFBC ASH COMPOSITIONS

	Chatham CFBC Bottom Ash	Chatham CFBC Fly Ash	TVA 20 MW Bag and Char Ash	TVA 20 MW Spent Ash	
				Sept.86 Study	Sept.88 Study
Chemical Composition (wt %)					
Moisture	0.03	0.12	0.24	0.01	0.17
Ash	99.21	93.22	81.13	97.59	95.70
Carbon (Total)	0.16	2.78	8.51	0.44	0.31
Carbon (Mineral)	0.13	0.52	0.60	0.30	0.16
Sulfur	12.20	6.35	8.24	14.45	12.80
Oxide Composition (wt %)					
SiO_2	10.36	18.40	15.48	4.25	4.54
TiO_2	0.16	0.28	0.30	0.11	0.07
Al_2O_3	3.13	5.64	6.67	1.49	1.36
Fe_2O_3	3.78	14.88	8.13	2.68	1.80
CaO	48.15	40.51	45.77	52.88	54.58
MgO	2.48	0.65	1.35	1.24	2.08
K_2O	0.45	0.84	0.85	0.18	0.20
Na_2O	0.20	0.53	0.54	0.07	0.28
P_2O_5	0.28	0.51	0.09	0.09	0.05
SO_3	30.50	15.88	20.61	36.13	32.00
CO_2	0.48	1.91	2.20	1.10	0.59
Forms of Calcium (wt %)					
CaO	51.85	26.97	22.51	26.17	32.18
$CaSO_4$	26.20	26.99	28.36	61.41	54.40
$CaCO_3$	1.08	4.33	5.02	2.54	1.33

TABLE 10 HYDRATION TESTS ON CHATHAM CFBC ASH

Mix*	H_2O Temp.(°C)	Ash Temp.(°C)	H_2O Added (%)	Peak Temp.(°C)	Suface H_2O	Chemically Combined H_2O	$Ca(OH)_2$ (wt %)
1	12	20	15	87	1.84	6.29	23.40
2	14	20	18	90	4.07	9.87	24.96
3	44	20	15	98	1.91	6.49	22.55
4	48	20	18	89	2.79	8.38	24.10
5	50	20	12	92	4.0	9.97	23.82
6	100	80	15	100+	2.06	7.19	19.27
7	15	20	20	100+	2.93	8.13	23.63

Theoretical Max. = 32.8% $Ca(OH)_2$

*All mix times were 5 minutes.

fly ash may restrict the amount of conversion possible without employing particle fracturing.

The testing program also included an investigation of the potential use of the Dalhousie PFA, a PFA locally available to the Chatham CFBC plant, as well as the Kingston PFA. The Kingston PFA performance was included to provide a comparison with the KECL research (2,4, and 5). For comparison, the chemical characteristics of the Dalhousie PFA are shown in Table 11, along with the composition of the Shawnee and Kingston PFA used in the KECL AFBC ash concrete studies. The most notable difference between the ashes was the high Fe_2O_3 content (about 39%) of the Dalhousie ash, which was reflected in the higher specific gravity of the Dalhousie ash (about 2.8) compared to the Kingston and Shawnee PFAs (about 2.4). All three PFAs are considered to be highly reactive pozzolans due to their behaviour in cement replacement and the Kingston PFA is presently commercially marketed by TVA as a pozzolan.

An investigation of the pozzolanic activity of the Dalhousie PFA was also conducted to compare performance of the Dalhousie PFA and to the Kingston PFA used in the KECL studies (Table 12). A comparison of the pozzolanic characteristics of the Kingston and Dalhousie PFAs is shown in Table 12. Both ashes meet the minimum requirements of ASTM C-618 (12) for a Class F pozzolanic fly ash. The pozzolanic activity index with $Ca(OH)_2$, or cement, was not determined, but previous testing (13) indicates that the Dalhousie PFA has good performance as a pozzolan. Subsequent testing of the Dalhousie PFA however has shown nearly 50% magnetic fraction predominantly composed of magnetite and little glass. The non-magnetic fraction of the Dalhousie PFA shows a considerably higher glass content typical of the Kingston PFA. It is generally believed that only the glassy fraction is active in the pozzolanic reactions. Van Roode *et al* (14) have determined glass contents (non-crystalline silicon and alumina compounds) of the Dalhousie PFA to be approximately 70%, with 22% magnetite and hematite. The Kingston PFA, on the other hand, is believed to have approximately 90% glass with quartz and mullite composing the majority of the remaining components. Although both the Dalhousie and Kingston PFAs are considered reactive pozzolans, the lower glass content coupled with the higher

TABLE 11 CHEMICAL AND PHYSICAL CHARACTERISTICS OF PULVERIZED COAL ASHES USED IN CHATHAM CFBC AND KECL AFBC ASH UTILIZATION STUDIES.

Chemical Composition (wt. %)	Shawnee	Kingston	Dalhousie
Moisture	0.35	0.38	0.09
Ash	93.10	96.33	97.88
Carbon	5.89	2.30	1.39
Sulfur	0.38	0.28	0.36
Loss on Ignition	6.77	3.42	1.54
Oxide Composition*			
SiO_2	54.51	51.20	39.41
TiO_2	1.75	1.40	0.71
Al_2O_3	32.19	30.58	13.52
Fe_2O_3	5.05	8.49	38.13
CaO	2.22	2.10	3.46
MgO	1.16	1.52	0.61
K_2O	2.19	3.13	1.82
Na_2O	0.51	0.32	0.31
P_2O_5	0.23	0.59	1.39
SO_3	0.19	0.18	0.46
	100.00	99.61	99.82
Specific Gravity	2.05	∿2.40	2.84

*750°C Ash

TABLE 12 SUMMARY OF POZZOLAN CHARACTERIZATION OF KINGSTON AND DALHOUSIE PFA.

	ASTM Class F	Kingston	Dalhousie
Parameters			
$SiO_2 + Al_2O_3 + Fe_2O_3$	>70%	90%	91%
SO_3	<5%	0.2%	0.5%
Moisture Content	<3%	0.4%	0.1%
LOI<12%	3.4%	1.5%	
Available Alkalis as Na_2O	<1.5%	NA	NA
Wet Sieve Plus 325 Mesh	<34	3.8	23.2
Pozzolan Activity with $Ca(OH)_2$	>800 psi	NA	NA

NA=Not Available

content of coarse-sized material suggest that the ash from Dalhousie may be less relatively reactive than that from Kingston.

The particle size distributions (PSD) of the Chatham CFBC fly ash and the Shawnee, Kingston and Dalhousie PFA ashes, are shown in Figure 6. All three ashes have a very fine PSD, having D_{50} of 25μm, 9.4μm and 19.5μm respectively.

It was proposed, for testing program, to use No. 57 limestone as coarse aggregate in the concrete mixes. Table 13 presents a comparison of the particle size distribution of the No. 57 limestone used in this study, and that used in the TVA study, with the requirements of ASTM C-33 (12). It is evident that the limestone used in this study meets all of the size distribution requirements under ASTM C-33.

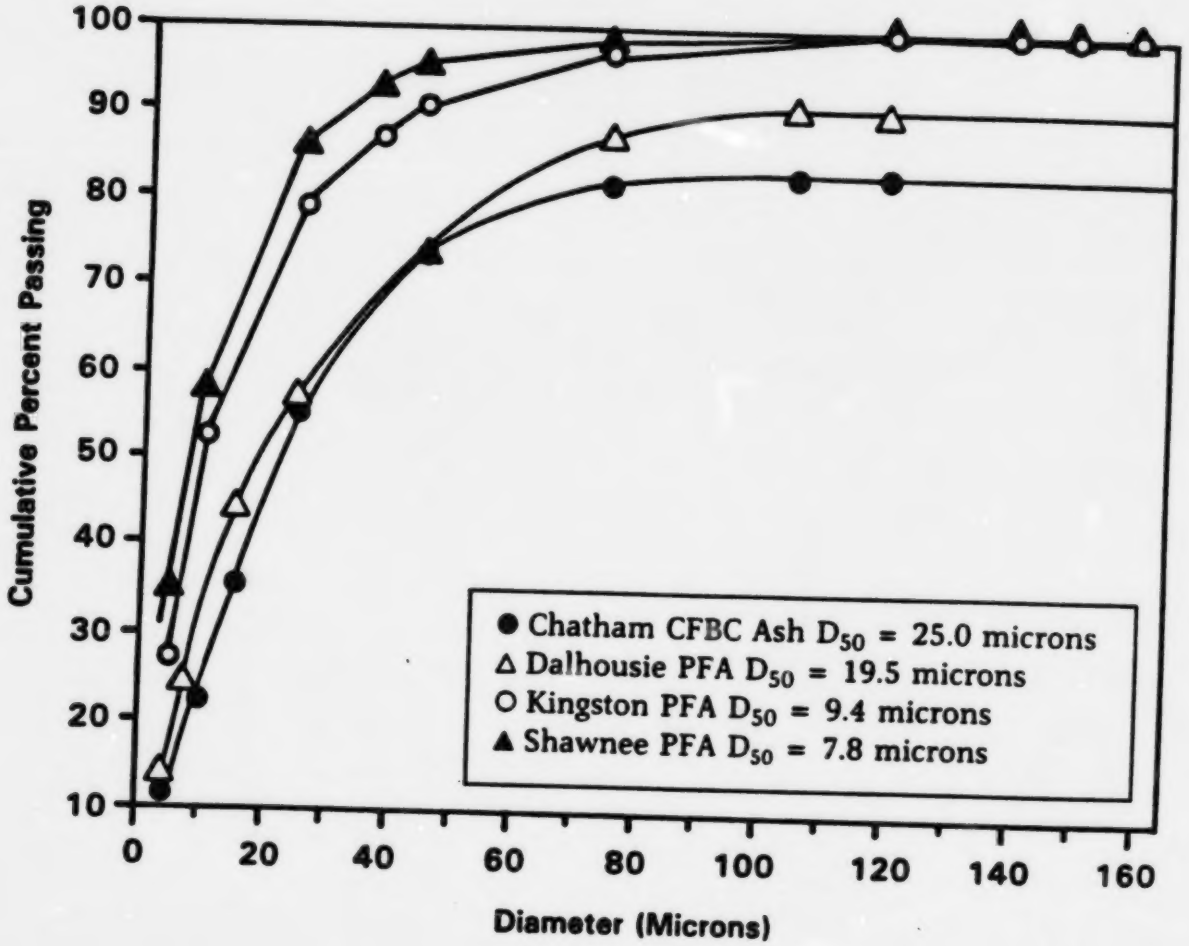


FIGURE 6 PARTICLE SIZE DISTRIBUTION OF CHATHAM CFBC ASH AND PULVERIZED FUEL ASHES USED IN THE STUDY

TABLE 13 COMPARISON OF THE PARTICLE SIZE DISTRIBUTION OF COARSE AGGREGATES USED IN CONCRETE MIXES

% Passing	Premier #57 Limestone Sept./88 Study	#57 Aggregate ASTM C-33	#57 Limestone TVA Study
1 1/2"		100	
1"	100	95-100	100
3/4"	95.5		84.9
1/2"	50.5	25-60	62.6
3/8"	31.7		
No. 4 Mesh	4.5	0-10	26.6
No. 8 Mesh		0-5	
Nominal		1" to No. 4	

NOTE: #57 Limestone Used in TVA Study was Slightly Finer than ASTM C-33 Specifications for #57 Aggregate.

4.0 MIX DESIGN/CONCRETE PERFORMANCE

The objective of the mix design testing was to define an "optimized" mix incorporating Chatham CFBC fly ash, Dalhousie PFA, lime (if required) and coarse (No. 57) limestone aggregate. No Portland cement was to be used in the mixes in order to effect a large cost incentive since Portland cement is the most costly component of conventional concrete. Previous studies (4, 5) have shown that strength development is dependent upon a variety of mix design parameters, several of which are significantly different for the Canadian ashes in comparison with the TVA ashes used previously namely:

- Potentially lower reactivity of the Dalhousie PFA
- Low lime availability of Chatham CFBC flyash
- Fine particle size of Chatham CFBC flyash and its effect on aggregate requirements

These were addressed in the mix design/concrete performance investigation.

4.1 Dalhousie PFA Reactivity

The reactivity of the Dalhousie PFA became of concern due to its relatively low glass content, the large amounts of magnetic material present and the high proportion of fine (plus 325 mesh) material. A series of mixes were produced to examine the extent of the difference in pozzolanic reactivity between the Dalhousie and Kingston PFAs. The mix proportions and strength development are shown in Table 14. It is clear that the Dalhousie PFA attained approximately 60% of the compressive strengths at 7 and 28 days observed for the same mix utilizing the Kingston PFA. However the mixes did use different PFA/$Ca(OH)_2$ ratios. Previous KECL testing (2,4, and 5) had shown that there was an optimum ratio above which compressive strengths decrease due to the diluent effect of the PFA and below which there was insufficient PFA for all of the available lime. More recent investigations now show that a rather large plateau in $Ca(OH)_2$/PFA ratio may exist. The KECL study had shown that the

TABLE 14 COMPARISON OF PERFORMANCE OF DALHOUSIE AND KINGSTON PFA USED IN CHATHAM AFBC MORTAR MIXES

Mix Proportions	wt %	Mix Proportions	wt %
Chatham CFBC Flyash	43.3	Chatham CFBC Flyash	44.6
Dalhousie PFA	15.8	Kingston PFA	12.8
Sand	23.7	Sand	24.4
Water	17.2	Water	18.2
Compressive Strength (MPa)		Compressive Strength	
7 days	3.64	7 days	5.6
28 days	11	28 days	18.9

optimum $Ca(OH)_2$/PFA ratio was approximately 1.15 by weight. Due to the differences in the specific gravity, and glass contents between the Kingston and Dalhousie PFAs, a $Ca(OH)_2$/PFA ratio of about 1.30 was used in this study.

4.2 Lime Availability

As was discussed above, after hydration, the Chatham CFBC fly ash was found to contain approximately 25% $Ca(OH)_2$, this being the result of hydration of only 76% of the total CaO present in the ash. By comparison, the spent bed residue from the TVA 20 MW combustor typically hydrated to produce 32 - 38% $Ca(OH)_2$. The poor strength development of the mixes containing the Chatham CFBC fly ash (Table 14) may have been caused by the lower quantity of available lime in this material.

To determine the extent to which lime availability was critical to strength development in AFBC no-cement concretes containing the Chatham CFBC fly ash, a series of mortar mixes was prepared (Table 15) using the Kingston PFA (already known to perform well with the TVA spend bed residue). To a basic mix (A-1), two levels of lime addition were made in proportions selected to maintain the PFA/$Ca(OH)_2$ ratio at 1.15 (mixes A-2 and A-3). A control mix (B-1) was prepared to confirm the effectiveness of the TVA spent bed residue in combination with the Kingston ash.

The strength developed by these mixes confirmed: (a) that lime addition, up to 8 to 9% improved the performance of the Chatham CFBC fly ash; (b) that addition of lime above this level was disadvantageous; and (c) that the TVA residue/Kingston PFA mix is a reactive system, without lime addition.

It was therefore concluded that there appears to be insufficient amounts of $Ca(OH)_2$ and PFA in the Chatham mixes to effectively bind the inert components of the mix, thus resulting in a lower strength for Chatham CFBC fly ash mixes.

A further series of mortar mixes was designed using the Dalhousie PFA, incorporated varying amounts of additional lime (Table 16). The mixes were tested

TABLE 15 EFFECTS OF LIME ADDITION ON UNCONFINED COMPRESSIVE STRENGTH OF CHATHAM CFBC ASH BASED CONCRETE MIXES

Mix Proportions (wt %)	A-1	A-2	A-3
Chatham CFBC Flyash	53.4	46.2	27.5
Lime	0.0	8.8	16.3
Kingston PFA	19.5	24.2	30.3
Water	27.1	20.9	25.9
Compressive Strength (MPa) 7 days	5.4	14.1	7.9

Mix Proportions (wt %)	B-1
TVA Spent Bed	56.8
Lime	0.0
Kingston PFA	25.1
Water	18.1
Compressive Strength (MPa) 7 Days	14.1

TABLE 16 EFFECT OF LIME ADDITION ON COMPRESSIVE STRENGTH OF CHATHAM CFBC FLY ASH MORTAR MIXES UTILIZING DALHOUSIE PFA

Mix Proportions (wt %)	D-0	D-1	D-2	D-3	D-4
Chatham CFBC Flyash	59.0	54.1	50.0	45.2	43.3
Lime	0.0	3.3	5.0	6.5	8.3
Dalhousie PFA	19.7	21.3	23.3	24.2	25.0
Water	21.3	21.3	21.6	21.0	23.3
Compressive Strength (MPa)					
7 days					
Humid*	4.6	3.9	4.3	3.9	2.7
Lime Sat**	3.7	3.1	3.6	3.3	3.3
28 days					
Humid*	11.5	9.9	11.3	11.5	7.8
Lime Sat**	7.8	8.0	10.1	9.5	8.3

*Cured in sealed container at room temperature.

** Cured in lime saturated water 23±2°C.

for compressive strength at 7 and 28 days and the resulting data are presented in Table 16 and Figure 7. It should be noted that the water/cement ratio for the mixes was maintained at approximately 0.60, which is rather high for good strength development. The 7 day strength test showed a decrease in strength with the amount of $Ca(OH)_2$ added, implying little benefit to strength development with increasing potentially reactive components. However, the 28 day strength data showed a slight improvement in strength with increasing $Ca(OH)_2$/PFA addition. $Ca(OH)_2$ additions of 8% or greater showed dramatic decrease in strength. The data also show a low strength 3.5 - 4.8 MPa to 10.3 - 11.7 MPa at 28 days. It is apparent that the reaction of $Ca(OH)_2$ with the Dalhousie PFA is slow and late strength development occurs. These data appear to agree with the work of Minnick (15) and Watt and Thorne (16), both of whom implied that low and late reactivity of high-iron fly ashes (such as the Dalhousie ash) are to be expected with lime.

4.3 Aggregate Requirements

This testing was designed to address, primarily, the fine aggregate requirements of AFBC concretes containing Chatham CFBC ash. During previous KECL studies, spent bed ash totally replaced fine aggregate in AFBC concrete due to the simlar PSD of spent bed ash and river sand. However, the Chatham CFBC ash has a very fine PSD. The ASTM PSD requirements for effective grading of fine aggregates are shown in Table 17 together with the PSDs for commercial concrete river sand and the Chatham CFBC fly ash and bed ashes. From these data it is clear that the Chatham CFBC flyash does not satisfy the fine aggregate requirements. The Chatham CFBC bed ash, however, does appear to have a PSD compatible with use as a fine aggregate. However, this ash represent only 3 - 5% of the total ash generated; and as a result, the combined CFBC ash would not meet the fine aggregate requirements of typical concretes. Additional testing to estimate the amount of fine aggregate that would need to be added to reach the desired grading characteristics was not conducted due to the already low amounts of reactive $Ca(OH)_2$ and PFA which are available in the mixes to bind the inert components.

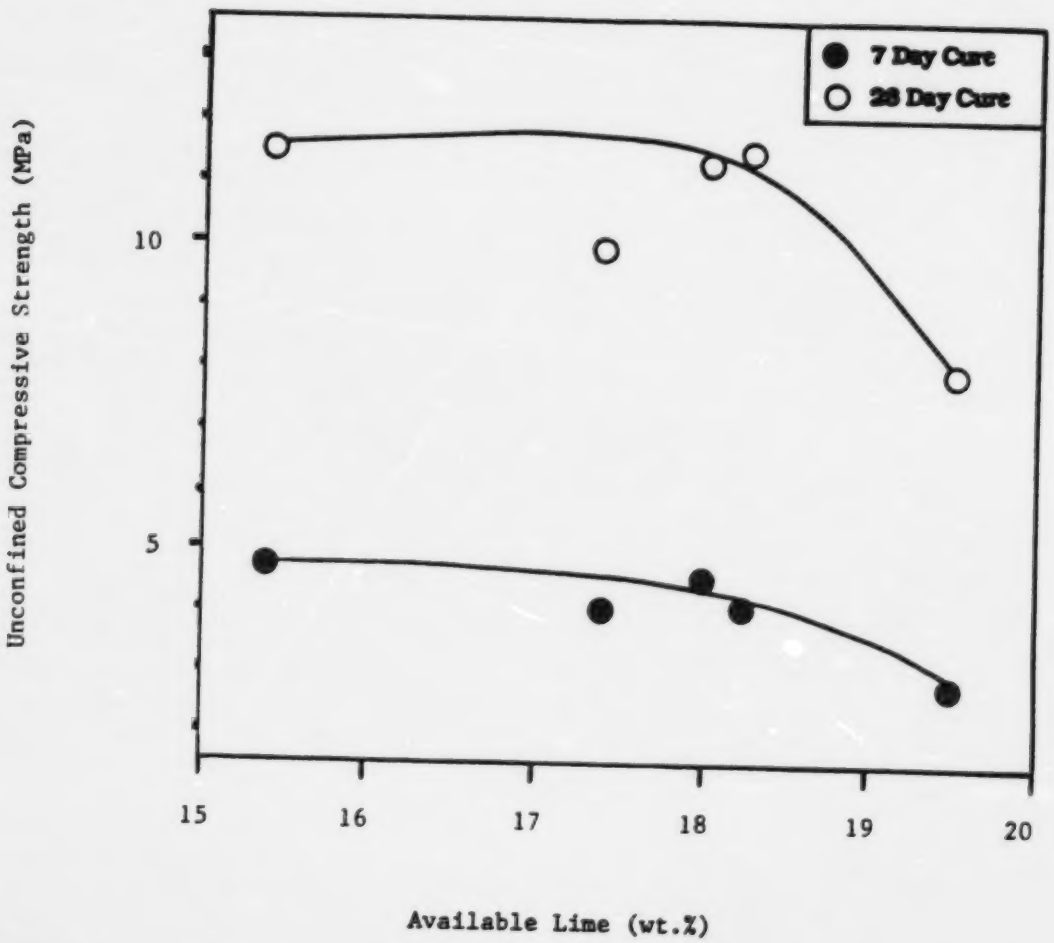


FIGURE 7 EFFECT OF AVAILABLE LIME CONTENT ON UNCONFINED COMPRESSIVE STRENGTH OF CHATHAM CFBC FLY ASH/DALHOUSIE PFA MORTAR MIXES.

TABLE 17 SIZE DISTRIBUTION AND AGGREGATE REPLACEMENT POTENTIAL OF CHATHAM CFBC ASHES

Cumulative % Passing Sieve	Commercial Concrete River Sand	ASTM C-33 Number 1 Concrete Sand	Chatham Oversize Bed Ash	Chatham Flyash
6 mm	100	100	100	
4 mesh	98	95 - 100	98	
8 mesh	83	80 - 100		
16 mesh	66	50 - 85	91	100
30 mesh	40	25 - 60		
50 mesh	10	10 - 30	37	98
100 mesh	2	2 - 10	11	94
200 mesh	1	-	2	84

However, a series of tests were conducted (Table 18) to examine the amount of coarse aggregate (No. 57 limestone) which could be incorporated in the mix design. The mixes were intended to maintain water/'cement' ratios of approximately 0.57. Workability was a major problem at these low water/'cement' ratios. Mixes G-1 and G-2, with low water contents and low aggregate contents formed 'balls' in the mixer, the nucleus of each ball being a coarse aggregate particle. Increasing the coarse aggregate content was expected to reduce the 'balling'. Two mixes were produced with coarse aggregate contents of approximately 50% (Table 18). This proportion of coarse aggregate was typical of the KECL AFBC no-cement concretes as well as conventional Portland cement concretes. In these two mixes (with and without lime addition), the water content was allowed to be increased in order to achieve a workability of approximately 25 mm slump. The water/'cement' ratios needed to achieve this level of workability were very high ranging from 0.65 to 0.92. The mix which incorporated lime addition had the lower water/'cement' ratio.

4.4 Performance Characteristics

Two concrete mixes (G-3 and G-4) using the Chatham and Dalhousie materials were produced in order to ascertain the performance characteristics of the 'optimum' mixes defined by the earlier testing. The mix proportions, the unconfined compressive strength data, setting time data, and unit weights are present in Table 19. The two mixes contained comparable coarse aggregate contents and the Dalhousie PFA/$Ca(OH)_2$ ratio was maintained at 1.29. Mix G-3 did not include additional lime, while Mix G-4 contained additional lime to a level of available lime equivalent to that found in the TVA spent bed residue used in previous AFBC no-cement concretes.

The compressive strengths of both mixes were low, achieving only 2.8 to 4.0 MPa at 28 days. It is believed that this low strength is a result of the

TABLE 18 EFFECT OF COARSE AGGREGATE CONTENT ON WATER/EFFECTIVE CEMENT RATIO AND WORKABILITY

Mix Proportions (wt %)	G-1	G-2	G-3	G-4
Chatham CFBC Flyash	41.7	35.7	27.4	18.6
Dalhousie PFA	14.1	12.1	9.1	11.8
Lime	0.0	0.0	0.0	3.9
No.57 Limestone Aggregate	30.0	40.0	48.6	52.3
Water	14.2	12.2	14.9	13.4
Water/Effective Cement	.57	.57	.92	.65
Workability Estimate	Balled (non-workable)	Balled (non-workable)	about 25 mm slump	about 25 mm slump

TABLE 19 PERFORMANCE CHARACTERISTICS OF PRELIMINARY CHATHAM CFBC CONCRETES

Mix Proportions (wt %)	G-3	G-4
Chatham CFBC Flyash	27.4	18.6
Lime	0.0	3.9
Dalhousie PFA	9.1	11.8
No.57 Coarse Aggregate	48.6	52.3
Water	14.9	13.4
Compressive Strength (MPa)		
7 Day Cure*	1.4	1.2
28 Day Cure*	2.8	4.0
Setting Time (hrs)		
Initial	15	18
Final	44	59
Unit Weight (mg/m^3)	2.20	2.19

* Cured in sealed container at room temperature.

high water/'cement' ratios required to achieve even a 25 mm slump. It is well known that a change of water/'cement' ratio of Portland cement concrete from 0.4 to 0.65 results in a reduction in 28 day compressive strength of 17.2 - 22 MPa. A further increase in water/'cement' ratio from 0.65 to 0.90 results in a further reduction in 28 day compressive strength to approximately only 6.9 MPa. Although the compressive strengths of the Chatham CFBC no-cement concretes are low, their high water/'cement' ratios are considered the major cause for this low strength.

The setting times of the two no-cement concretes were also determined according to ASTM C-403 (12). The results, presented in Table 19, indicate that an extended time is required to achieve both initial and final setting. Figure 8 presents the setting time data for the Chatham CFBC no-cement concrete and conventional Portland cement concrete. Both the initial and final setting times were extended, with final setting times in excess of 58 and 44 hours for the Chatham CFBC no-cement concretes with and without lime addition, respectively. This compares to a final setting time of about 7 hours for Portland cement concrete and about 24 hours for the KECL AFBC no-cement concrete. This lengthening of setting times is believed to be the result of high levels of PFA and the high water requirements. It is also believed that these setting times could be improved dramatically by the use of chemical accelerators.

The unit weight for the Chatham CFBC no-cement concretes (Table 19) are low and are typically around 2.2 mg/m^3. This compares to typical unit weights of 2.42 mg/m^3 for Portland cement concretes. The low unit weights are a result of the lower specific gravities of the Chatham CFBC flyash and the high water requirements of these CFBC no-cement concretes.

4.5 Summary

In summary, the results of the program testing have demonstrated that there are significant differences in bubbling and circulating FBC ashes and their performance in no-cement concretes. Most notable are the lime availability

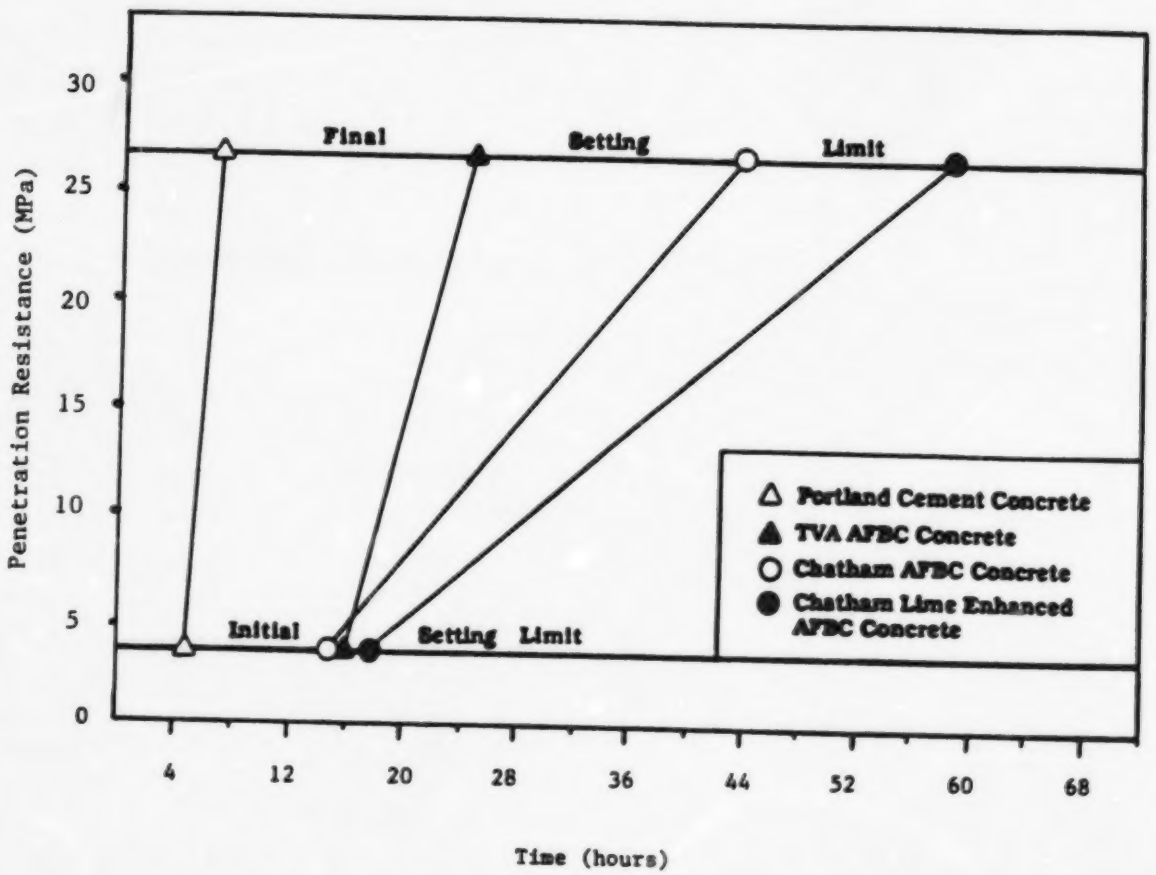


FIGURE 8 COMPARISON OF SETTING TIMES OF CONVENTIONAL PORTLAND AND AFBC NO-CEMENT CONCRETES

and very fine particle size distribution. These result in low amounts of cementious components and poor workability. Compressive strength development and setting times are therefore adversely affected due to the need for high water/'cement' ratios and high PFA concentrations.

The design and performance of AFBC no-cement concretes appears to also be very dependent upon the characteristics of PFA used. Whereas, the highly glassy, high silicon and aluminum Kingston PFA results in good strength development, the relatively low glass, coarse, high-iron Dalhousie PFA results in lower ultimate strength and poor early-strength development.

5.0 CONCLUSIONS

As was discussed in a previous report, (4) work at KEC showed that concrete, containing no Portland cement or natural sand, can be produced from suitable combinations of AFBC residue and pulverized fuel fly ash (PFA). Using a combination of residue from the TVA Shawnee 20 MW AFBC combustor and PFA from the Kingston Generating Station, concrete mixes were prepared (Table 20) that developed approximately 27.6 MPa compressive strength after 28 days of curing. On the basis of these findings it can be concluded that the use of FBC residues has the potential for commercial production of concretes suitable for some structural applications. Such applications would prove attractive not only in terms of waste utilization but also as a means to obtain cost savings in some concrete works. To illustrate the potential value of the materials for use in concrete, Table 21 (17) has been prepared showing typical concrete and concrete-making material costs in Atlantic Canada.

The work conducted during this project was directed to determine: (a) whether concretes similar to those made by KEC can be prepared from available Canadian materials; (b) the potential for commercial utilization of such concretes. The materials studied were a circulating FBC residue from the combustor at Chatham and a PFA from the Dalhousie Generating Station both in New Brunswick.

The results of the experimental work on these materials, indicate that they do not offer the same potential for the production of structural concretes in simple combinations as do those previously studied by KEC. In particular, both the rate of strength development and the ultimate strength of the concretes produced from the Chatham/Dalhousie material combinations were much lower than from the TVA/Kingston mixes.

It appears based on the data available from this study, there are at least two factors involved in determining the properties of 'no-cement' concretes:

(1) the particle size distribution of the AFBC residue;
(2) the reactivity of both the AFBC residue and the PFA.

TABLE 20 PROPORTIONS OF MATERIALS USED TO PREPARE NO-CEMENT CONCRETES (4)

Component	Percent by Mass	Percent by Volume
PFA	10.9	11.5
Spent FBC Bed	29.4	24.1
Coarse Aggregate	48.3	38.7
Water	11.4	24.7

TABLE 21 TYPICAL COSTS FOR CONCRETE AND CONCRETE-MAKING MATERIALS IN ATLANTIC CANADA (1988) (17)

Material	Typical Cost ($Can)
Ready-mixed Concrete:	
20 MPa	$ 78/m^3
35 MPa	$ 90/m^3
Aggregates:	
Sand	$ 7/tonne
Stone	$ 6/tonne
Cement:	$130/tonne
Fly ash (PFA)	$65 - 70/tonne

In the case of the Canadian materials both factors appear to work against the development of strength in mortar and concrete mixes for reasons discussed briefly below.

The influence of particle size distribution relates largely to its effect on the quantity of water required to render the mixes fluid enough to be workable. Normal concrete contains aggregate particles that range over a broad gradation of sizes such that much of the space between the large aggregate is filled by sand. Experience has shown that by controlling the aggregate gradation of concrete mixes it is possible to achieve a minimum the water requirement. When no admixtures are used, this is usually found to be at a level related to the cement content of the mix, such that a water/cement ratio (w/c) of approximately 0.5 to 0.3 is required. In normal concretes w/c is approximately representative of the overall water/fine particle ratio, since most fine-sized particles are Portland cement. The resulting concretes are porous to the degree that more water is added to the mix than is required to hydrate the Portland cement. The excess water forms void space in the hardened mass. However, such mixes will develop strength to about 20 to 34 MPa after 28 days.

In the case of a mixture containing no sand to fill the space between large aggregate particles, most of the volume of the mix comprises fine, silt-sized particles. These have a high suface area and can absorb large volumes of water before sufficient free water is available to act as a continuous medium separating the particles from each other, this being the condition required for workability. As a consequence, any water not required for hydration, will cause excessive void formation in the concrete mass and result in low compressive strength. If, in addition, the rate or extent of hydration is low, the binding of the mass will be reduced and the void space will be increased. The result will be concrete of exceptionally low compressive strength.

The TVA/Kingston mixes studied by KEC contained sand sized particles from the AFBC residue and had a water/fine particle ratio of approximately 0.28 (4).

With the Chatham/Dalhousie 'no-cement' concretes, in order to obtain approximately 25 mm of slump (Mix G-3), it was necessary to raise the water/fine particle ratio to 0.41. If all of the fine particulate material was cementitious and formed hydrated binder, such a w/c ratio should lead to an acceptable concrete. Thus, in this case it is concluded that, not only was a relatively large water demand induced by the particle size characteristics of the FBC residue, but many of the AFBC residue and the PFA particles were insufficiently reactive to participate in binder formation. Strength development was thus reduced in two ways: (a) large void-volume formation on hydration; and (b) low binder content in the hydrated mass.

The TVA/Kingston material combination appears to react to form cemented material in a period comparable to normal Portland cement. Thus mortars of approximately 13.8 MPa at 7 days were made. In the absence of lime, a combination of the Chatham AFBC residue and the Kingston ash was much less reactive (5.4 MPa at 7 days). When lime was added to the mixture, reactivity was enhanced and a mortar of 14 MPa was produced at 7 days. In addition to the lower reactivity of the Chatham residue, there is some indication that the Dalhousie PFA is relatively less reactive than the Kingston ash. Thus the problem of low strength development is exacerbated in Chatham/Dalhousie mixes.

In terms of utilization of the Canadian materials the following conclusions may be drawn:

(1) "No-cement" concretes suitable for structural applications cannot be made from simple combinations of the Chatham and Dalhousie wastes.

(2) Some improvement in strength development of mixes containing Chatham CFBC ash and Dalhousie PFA can be obtained by optimal addition of lime, though it is not likely that strengths necessary for structural applications can be attained.

(3) Concretes suitable for underground backfill, controlled-density fill type materials, or roller compacted concrete applications can probably be made from these wastes.

(4) It is considered unlikely that any PFA will be found that is reactive enough to compensate for the high water demand induced by the fine particle size characteristics of the Chatham residue. Other CFBC residues may be similarly limited due to fine particle sizes.

(5) It must be concluded that the potential for production of no-cement concretes for other than low-grade applications will be very much influenced by source specific factors, such as particle size distribution.

In light of the above considerations the following recommendations are made:

(1) The Chatham AFBC residue, in combination with any available fly ash should be examined as a potential backfill material or as a component of a controlled density fill (eg. K-Crete).

(2) Only one CFBC residue/PFA combination has been investigated to date for the production of no-cement concrete. A broader range materials from different combustors using different coal/limestone combinations under different operating conditions should be studied to better define the practical limits of this utilization option.

(3) The nature of the hydration and bonding in AFBC/PFA combinations should be examined to establish whether chemical or physical methods to accelerate hydration or reduce water demand are possible.

(4) A more intensive investigation of the successful TVA/Kingston mixes is justified to establish the extent to which they are dependent on the specific characteristics of the TVA residue.

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